# *(Water Supply)*

**Forecasting population:-**

1-Arithmetic method; rate of growth is constant

$dp\ / dt = k\ , k = \Delta p/\Delta t$

∴population in the future is : $P_t$

$$P_t = P_0 + k\Delta t$$

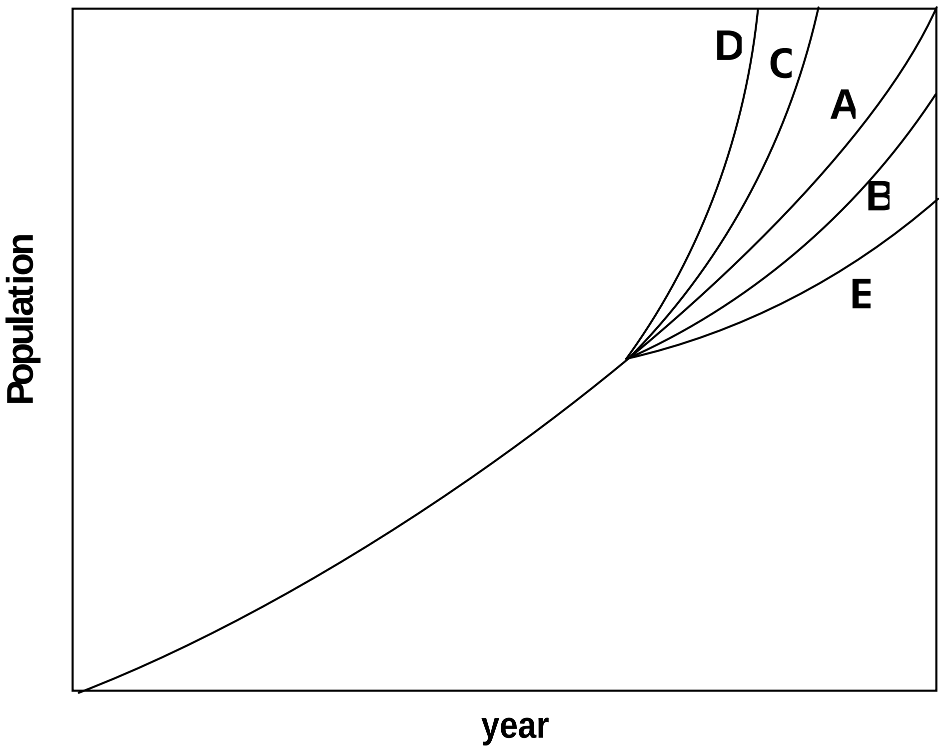
2-uniform percentage method ; rate of increase is proportional to 34

$$\frac{dP}{dt} = k^{/}p$$

$$\ln p = \ln p_{0+}\ k^{/}\Delta t$$

$$\therefore k^{/} = \frac{\ln p - \ln p_0}{\Delta t}$$

3-curvilinear method



Declining growth method ; rate of growth is a function of its population deficit $\frac{dp}{dt} = -k''(p_{sat} - p)$

$$k'' = -\frac{1}{n}\ln\frac{p_{sat} - p}{p_{sat} - p_0}$$

p,$p_0$:poplation in (n) years

$$\therefore future\ population \quad p = p_0 + (p_{sat} - p_0)(1 - e^{k''\Delta t})$$

Water consumption for various purposes:

Danmastic :300L/capital/day.

Industrial :160 L/capita /day.

Commercial:100L/capital day.

Loss and waste : 50L/capital /day.

∴ total :670L/capital/day.

Factors Affecting consumption :

Size of city , presence of industries , quality of the water , its cast , its pressure , the climate , characteristics of the population , meters ,and efficiency of the water works administration .

*max daily consumption =1.8 anneals mean .

Fire demand;

$F=18c(A)^{0.5}$ $\quad$ $F=9p$ m ,$A=ft^2$

$F=3.7c(A)^{0.5}$ $\quad$ F=L/s ,$A=m^2$

Minimum 500 gpm (1890L/min ,32.2L/sec).

Gallons =liters %0.264 , $m^2=10.76ft^2$ or gall =3.78 liters .

C= constant (0.6-1.5 ), 0.6min , 1.5 max .

Ex :population 22000 cap ,consumption 600L/cap ./day five for building $1000m^2$, 6 stories ,

Solution :Av. Domestic Demand =22000cap.× 600L/cup/day consumption 600L/cup/day

$$=13.2\times10^{6} L/day .$$

*max . daily demand =1.8 ×Av. = $23.76\times10^{6}$L/day .

$F=18\ (1)(1000\times10.76\times6)^{0.5}=4574gpm.$

4574 gpm ×3.78L/gall =17288.8 L/min =$24.89\times10^{6}$L/day

∴ max =$23.76\times10^{6}+24.89\times10^{6}=48.65\times10^{6}$L/day.

$48.65\times10^{6}$L/day /22000cap =2211L/cap/day .

If we need 10 hr . only for fire demand :

∴total flow =$23.76\times10^{6}+24.89\times10^{6}\times\frac{10hr}{24hr/d}$

$$=34.13\times10^{6} L/day$$

$34.13\times10^{6}$/22000=1551L/cap/day.

***<u>Table (2-3)Residential five flows</u>***

| Distance between adjacent units | | Required fire flow | |
|---|---|---|---|
| ft | m | gpm | L/min |
| >100 | >30.5 | 500 | 1890 |
| 31-100 | 9.5-30.5 | 750-1000 | 2835-3780 |
| 11-30 | 3.4-9.2 | 1000-1500 | 3780-5670 |
| ≤10 | ≤ 10 | 1500-2000 | 5670-7560 |

***Table (2-4)Fire flow Duration***

| Required flow | | Duration ,hr |
|---|---|---|
| gpm | L/min | |
| <1000 | <3780 | 4 |
| 1000-1250 | 3780-4725 | 5 |
| 1250-1500 | 4725-5670 | 6 |
| 1500-1750 | 5670-6615 | 7 |
| 1750-2000 | 6615-7560 | 8 |
| 2000-2250 | 7560-8505 | 9 |
| >2250 | >8505 | 10 |

Design Periods:

Source 50 years, pipe line from source 25 years water treatment plant 10-15 year, pumping plants 10 years , Distribution system 10-15 years .

Maximum Ratio Rate :

$$M = 1 + \frac{14}{4 + p^{1/2}}$$

M =max / average ratio .

Corrosion:

Is the conversion of amatol to salt or oxide with a loss of desirable properties such as mechanical strength .

In corrosion electron transfer must occur either in similar metal or different metal .zone releases electron is anodic , accept them called catholic.

Anode Fe $\longrightarrow$ $Fe^{++}$+ 2 electron

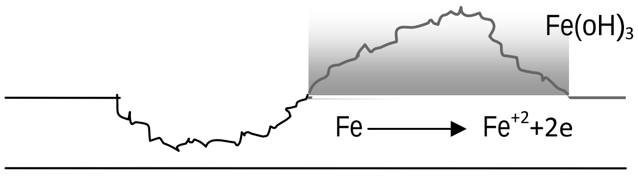
Cathode 2electron +$H_2o$+1/2$O_2$ $\longrightarrow$ $oH^-$

Or. $2H^+$+1/2$O_2$+2e $\longrightarrow$ $H_2O$ Cathode reaction .

$Fe^{+2} + 2OH^{-} \longrightarrow Fe(OH)_2$ Anode reaction.

$Fe(OH)_2 + O_2 + 2H_2O \longrightarrow Fe(OH)_3$ Anode reaction.

∴ Metal dissolution is the result of iron oxidation at the anode coupled with reduction of oxygen at the cathode .




Or ferrous $Fe^{++}$ iron $\xrightarrow{\text{Oxidized}}$ to $Fe^{++}$ (ferric)in presence of $O_2$ and precipitates as ferric hypoxic or rust :

$2Fe^{++} + 5H_2O \longrightarrow 2Fe(OH)_3 \downarrow + 4H$ .

Rate of corrosion α. And diffusion rate of dissolved oxygen to surface of iron .

Temperature variation , dissolved salts and presence of micro organisms influence corrosion . chloride ions accumulate at anode .

<u>Protection Against Corrosion :</u>

1-cathodic protection :is supplying aDC voltage so that electron will flow in to it at least equal to that at when they left under conditions of corrosion .

2- coating :paint , cool bar ,asphalt ,epoxy ,cement .

3- Metallic coating.

<u>Hardy cross method of Analysis of Flow in pipes :</u>

$\sum Qi = \sum qi \; , h = \frac{v^2}{2g} \; , Q = va$

The manning ,chezy, and hazen williams formulas are used , they can be expressed in general terms as :

$h = KQ^{X}$ , h= Head loss in pipe , k=constant ,Q=discharge .

$h = kQ^{1.85}$ ,Q=Q+Δ ,Q=actual amount , $Q_1$=assumed , Δ=correction .

$KQ^X = K(Q_1 + \Delta)^X = K[Q_1^X + XQ_1^{(X-1)}\Delta + \cdots]$

$\sum KQ^X = 0$

$$\therefore \Delta = -\frac{KQ_1^X}{XKQ_1^{(X-1)}} = \frac{-h}{K\sum h/Q}$$

<u>The procedure can be expressed as follow:</u>

1-Assume any initially consistent distribution of flow .

$$\sum Q_{inlet} = \sum q_{out\ let}$$

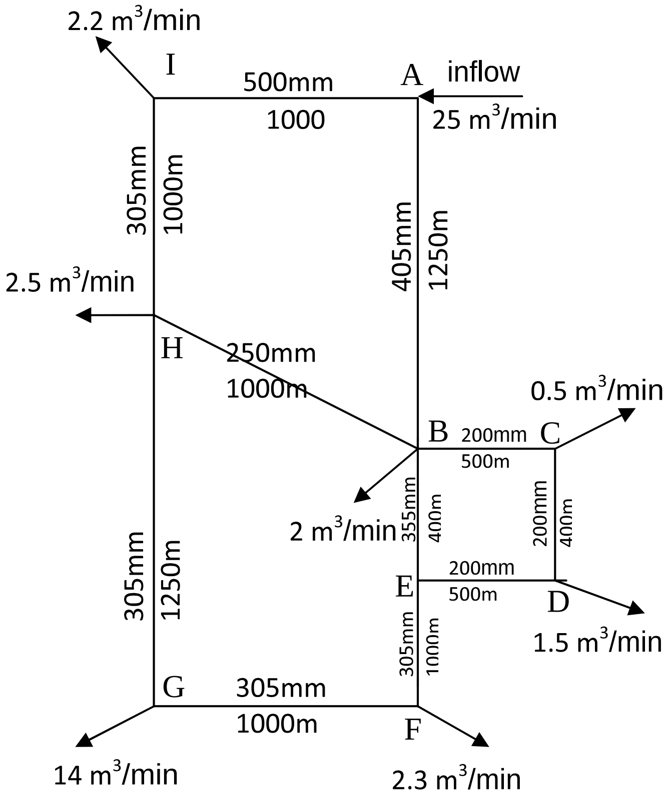
2-Compute the head loss in each pipe by the diagram p.118 .

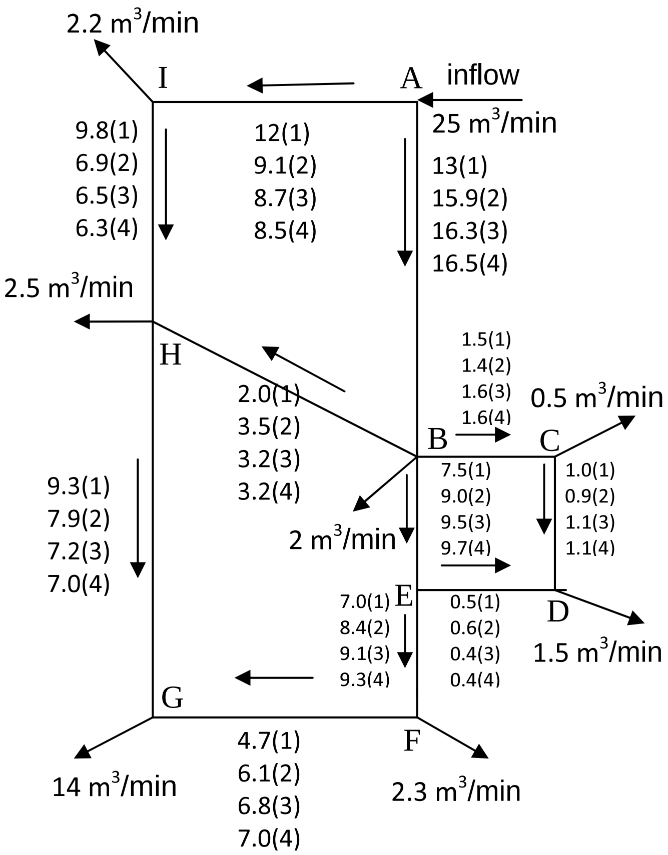
3- compute the head for each pipe length and the sum of them in each circuit (loop).

4-computer the head –discharge ratio and the sum of them in each circuit (loop ).

5-apply the correction equation $\Delta = -\sum h/k\sum h/Q$

Example :


2.2 m3/min
I
500mm
1000
A
inflow
25 m3/min
305mm
1000m
405mm
1250m
2.5 m3/min
H
250mm
1000m
0.5 m3/min
B
200mm
500m
C
2 m3/min
355mm
400m
200mm
400m
305mm
1250m
E
200mm
500m
D
1.5 m3/min
305mm
1000m
G
305mm
1000m
F
14 m3/min
2.3 m3/min


2.2 m3/min
I
A
inflow
25 m3/min
9.8(1)
6.9(2)
6.5(3)
6.3(4)
12(1)
9.1(2)
8.7(3)
8.5(4)
13(1)
15.9(2)
16.3(3)
16.5(4)
2.5 m3/min
H
1.5(1)
1.4(2)
1.6(3)
1.6(4)
2.0(1)
3.5(2)
3.2(3)
3.2(4)
0.5 m3/min
B
C
9.3(1)
7.9(2)
7.2(3)
7.0(4)
7.5(1)
9.0(2)
9.5(3)
9.7(4)
1.0(1)
0.9(2)
1.1(3)
1.1(4)
2 m3/min
E
7.0(1)
8.4(2)
9.1(3)
9.3(4)
0.5(1)
0.6(2)
0.4(3)
0.4(4)
D
1.5 m3/min
G
F
4.7(1)
6.1(2)
6.8(3)
7.0(4)
14 m3/min
2.3 m3/min

Hardy cross Aalysis:
First correction
Loop I

| Line | Flow $M^3$/min | Dia. m | Length m | S | h. m | h/Q $m/m^3$/min |
|---|---|---|---|---|---|---|
| AB | 13 | 0.40 | 1250 | 0.0110 | 13.75 | 1.058 |
| BH | 2 | 0.25 | 1100 | 0.0033 | 3.63 | 1.815 |
| HI | -9.8 | 0.30 | 1000 | -0.0260 | -26.00 | 2.653 |
| IA | -12.0 | 0.30 | 1000 | -0.0380 | -37.8 | 3.150 |
| | | | | | -46.42 | 8.676 |

$\Delta_I = \frac{-46.42}{1.85(8.676)} = 2.9$

Loop II

| Line | Flow $M^3$/min | Dia. m | Length m | S | h. m | h/Q $m/m^3$/min |
|---|---|---|---|---|---|---|
| BE | 7.5 | 0.35 | 400 | 0.0075 | 3.00 | 0.400 |
| EF | 7.0 | 0.35 | 600 | 0.0066 | 3.96 | 0.566 |
| FG | 4.7 | 0.30 | 1000 | 0.0067 | 6.68 | 1.423 |
| GH | -9.3 | 0.30 | 1250 | -0.0236 | -29.54 | 3.177 |
| HB | -2.0 | 0.25 | 1100 | -0.0033 | -3.63 | 1.815 |
| | | | | | -19.53 | 7.381 |

$$\Delta_{II} = -\frac{-19.53}{1.85(7.381)} = 1.4$$

Loop III

| Line | Flow $M^3$/min | Dia. m | Length m | S | h. m | h/Q $m/m^3$/min |
|---|---|---|---|---|---|---|
| BC | 1.5 | 0.20 | 500 | 0.0058 | 2.91 | 1.937 |
| CD | 1.0 | 0.20 | 400 | 0.0028 | 1.10 | 1.110 |
| DE | -0.5 | 0.20 | 500 | -0.0008 | -0.38 | 0.762 |
| EB | -7.5 | 0.35 | 400 | -0.0075 | -3.00 | 0.400 |
| | | | | | 0.63 | 4.209 |

$\Delta_{III}$=-$\frac{0.63}{1.85(4.209)} = -0.1$

Hardy cross Aalyss

second correction

Loop I

| Line | Flow $M^3$/min | Dia. m | Length m | S | h. m | h/Q $m/m^3$/min |
|---|---|---|---|---|---|---|
| AB | 15.9 | 0.40 | 1250 | 0.0157 | 19.65 | 1.236 |
| BH | 3.5 | 0.25 | 1100 | 0.0094 | 10.34 | 2.954 |
| HI | -6.9 | 0.30 | 1000 | -0.0136 | -13.60 | 1.971 |
| IA | -9.1 | 0.30 | 1000 | -0.0227 | -22.70 | 2.495 |
| | | | | | -6.31 | 8.656 |

$\Delta_I$=0.4

Loop II

| Line | Flow $M^3$/min | Dia. m | Length m | S | h. m | h/Q $m/m^3$/min |
|---|---|---|---|---|---|---|
| BE | 9.0 | 0.35 | 400 | 0.0105 | 4.20 | 0.467 |
| EF | 8.4 | 0.35 | 600 | 0.0093 | 5.58 | 0.664 |
| FG | 6.1 | 0.30 | 1000 | 0.0108 | 10.80 | 1.770 |
| GH | -7.9 | 0.30 | 1250 | -0.0175 | 21.88 | 2.769 |
| HB | -3.5 | 0.25 | 1100 | -0.0094 | -10.34 | 2.954 |
| | | | | | -11.64 | 8.624 |

$\Delta_{II} = 0.7$

Loop III

| Line | Flow $M^3$/min | Dia. M | Length m | S | h. m | h/Q $m/m^3$/min |
|---|---|---|---|---|---|---|
| BC | 1.4 | 0.20 | 500 | 0.0051 | 2.55 | 1.821 |
| CD | 0.9 | 0.20 | 400 | 0.0023 | 0.92 | 1.022 |
| DE | -0.6 | 0.20 | 500 | -0.0011 | -0.55 | 0.917 |
| EB | -0.9 | 0.35 | 400 | -0.0105 | -4.20 | 0.467 |
| | | | | | -1.28 | 4.227 |

$\Delta_{III}$=O.2

Ex: city population 2800 capita in 2000 if each capita need 1/350 gall ,in which period will the population being 35000 cap. ,and if this city needs $16\times10^6$ L/d in year 2000 ,total capacity $5\times10^6$ gall/day ,what the period need to arrive far this capacity .

$P_t=p_0+k\Delta t$ ,$k=\frac{cap}{gall}=\frac{1}{1/350}=350\frac{cap}{gall}$

$35000=28000+350\Delta t$

$\Delta t$=20 year

$\frac{16\times10^6 L/day}{28000}$=571L/cap/day

Capacity =$5\times10^6$ gall/day ×3.78=$28000+350\Delta t$

Δt=14.3 year.

Ex(2):determine the fire flaw required far residential area consisting of homes of ordinary construction ,2500$ft^2$in area 10ft apart .what the total volume of water must be provided to satisfy the fire demand of this area

Solution :-$f=18C(A)^{o.5}=18(1)(2500)^{0.5}$=900gall/min (gps)

But 900gpm <1500gpm

From table (2-3) far distance 10ft =3.0m

Required fire flow =1500-2000gpm =5670-7560L/min , ∴ take F=1500gpm

From table (2-4)

This is tasting for 6hr =360 min

∴ total volume =1500gpm×360 min

=540000gallon ×3.78 L/gallon .

$=5.81\times10^6 m^3$.

EX: determine the required fire flow 3-storywood frame building covering $700m^2$which connects with α 5-story building of fire resistance construction covering $900m^2$.

Solution :total area =3×700 +5×900 =$6600m^2$

$\frac{3\times700}{6600}$ =0.32

$F1 = 3.7\times 1.5\times(6600)^{0.5} =$ , $F_2 = 3.7\times0.6(6600)^{0.5} =$

$\frac{fire\ resistance\ constr.}{total\ area}=\frac{5\times900}{6600}$=0.68

$\therefore Ftotal = 3.7[1.5\times(0.32(6600))^{0.5} + 0.6\times(0.68(6600))^{0.5}]$

=266.92L/sec

403.2L/s

***<u>Pumps and pumping stations :</u>***

Pw=KQH

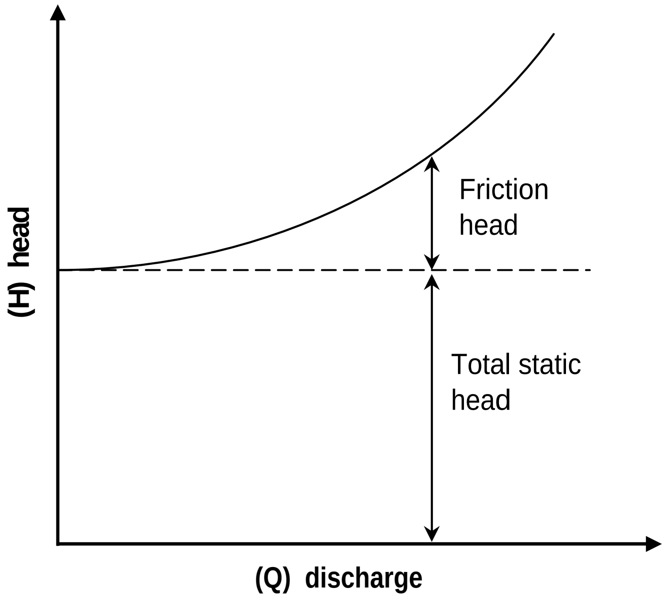
Pw=the power required .

Q=flaw$m^3$ /min ,gpm .

H=total head m,ft .

K=constant dependent puam

Fluid density and water temp.




***System Head Curve***

$=2.52\times10^{-4}$ (hp, gpm , ft )=0.163

(kw ,m$^3$ ,/min ,m)

Hp=0.7457 kw ,Efficiency = $\frac{Pout(kw)}{Pinput(kw)}$ center fugal pumps

$\frac{Q_1}{Q_2} = \frac{N_1}{N_2}$ Where:$Q_1$,$Q_2$ =discharge liter/sec

$\frac{H_1}{H_2} = \frac{N_1^2}{N_2^2}$ $H_1$,$H_2$=Head meter

$\frac{P_1}{P_2} = \frac{N_1^3}{N_2^3}$ $N_1$,$N_2$= Number of revolution perm in (rpm)

$\frac{Q_1}{Q_2} = \frac{D_1}{D_2}$ $p_1$,$p_2$=power (kw)

$\frac{H_1}{H_2} = \frac{D_1^2}{D_2^2}$ $D_1$,$D_2$=Impeller diameter(m)

$\frac{P_1}{P_2} = \frac{D_1^3}{D_2^3}$

***Pumps Type:***

a- Volute pump.
b- Open impeller pump.
c- Closed impeller pump.
d- Closed impeller with double section pump.
e- Turbine pump.

EX: Determine the motor power far pump system design to deliver 1.89 m$^3$/min (500gpm) against total system head 50m (164ft) .assume the efficiency of bath pump and motor 80% .

Solution :-

$P_w$=KQH=0.163×1.89×50

=15.4×1000÷ 745.7

=20.7hp.

$P_{out\ put} = \frac{15.4}{0.8} = 19.25KW$

$P_{INPUT} = \frac{19.25}{0.8} = 24.06KM$

## ***Quality Of water supply***

Impurities of water :

1-micro organisms: cause disease.

2-Alagae :cause disagreeable odor .

3- certain salts :cause unp lea set tastes , hardness , corrosiveness .

4-Gases :cause ador and corrosiveness .

*commericable diseases : which may be transmitted by water include bactericidal ,viral and protozoan infection .

Bacteria: size range between 0.5-5Mm visible through microscope ,may be spheres , rods or spirals shaped may be appear singly ,in pairs packets or chairs .

Reproduction by binary fission every 15-30 min ,oxygen and nutrient ,include :typhoid ,salmonellas is, bacillary dysentery.

contaminate water :is one that contain microorganisms, chemical , industrial or other wastes or sewages so that it is unfit for intended use .

viruses :size 20-100 nana meter ($nm=10^{-9}$) replicate only in living hosts cell , faunal in human feces ,or infected person transmitted by fecal cause infection hepatitis and polio my elitist .

protozoan: smallest type 10-50Mm, multiply by binary fission cause amoebic mesentery and giardiasis G-G rotifers.

Determination of bacterial number :

paper , water pass , bacteria rinse filter place it upturn nutrient media M-Endo broth incubate at $35c^0$ far (20-22) hr. for coli form (fecal cal. 44.5 – far 22 hr)

2-multiple tube fermentation :

Put small size sample several tubes containing nutrient contain a lactose incubates far 24hr . at $35c^0$, if gas present is positive test ,if no gas present is negative test .

MPN :most probable number .

Find by using statistics .

Tables which are available .

Turbidity :

passage of light through the water in which visual depth is restricted .

Turbidity may be caused by a wide variety of suspended materials which rang in size prom colloid 1Mm to coarse dispersion depending upon the degree of turbulence .

It is measured by instrument , is the in testing of light scaffored by turbidity partials

NTU : nophlo metric turbidity unit .

Standard for drinking water supply <5NTU

Color : is caused by material in solution or colloidal state and should be distinguish from turbidity may cause exponent color .

Color caused by suspended matter is referred as apparent color .color due to vegetable organic extracts that are colloidal called true color .

The color expressed by color unit and measured by compression to a platinum cobalt standard color unit .

Alkalinity of water :-

Its a measured of its capacity to neutralize acids . the carbonates (ca $co_3$)and bicarbonates(ca($HCO_3$)) of calcium ,sodium and magnesium are major from of calcium , sodium and magnesium are major from of alkalinity .

Units are mg/L as $caco_3$ in term of equivalent calcium carbonate

Acidity :-is caused by carbon dioxide or by strong mineral acids .

Acidity is measured in terms of the calcium carbonate needed to neutralize the carbonic acid expressed in mg/L as $caco_3$.

PH:- is measured of the concentration of free hydrogine ion in water .

$$HOH \rightleftharpoons H^+ + OH^-$$

$$K_W = \frac{[H^+][OH^-]}{[HOH]}$$

$K_W = [H^+][OH^-]$, $[H^+]=[OH^-]=10^{-7}$mol/L

$K_w=10^{-14}$ at $20C^0$

$\therefore \log[H^+]+\log[OH^-] = \log 10^{-14}=14, \therefore log = P$

$[H]$=[oH]

PH=-log $[H^+]$,[H] $=10^{-4}$mol /L

PH=4

Soluble mineral impurities :

Carbonate and bicarbonate of calcium and magnesium soluble in pure water ,but when carbon dioxide is present , it will dissolve freely to from bicarbonates temporary hardness : can be removed by boiling the water it will precipitate to from scale in kettles ar steam boilers .

On carbonate hardness :

Is caused by the sulfate and chloride salts of the divalent cantinas .

It is not altercated by boiling and is called permanent hardness .

**Treatment of water –clarification :**

Screens : they serve as protective device for the reminder of the plant rather than as treatment process . harge bulk objects can be removed by bar screens: consist of vertically placed bars with openings 25 mm (1 in) see fig .(9-1) .

Fine screen with openings =6mm (1/4in) removes leaves twinges .

See fig (9-2),bar screen cleans manually or mechanically approach velocity 0.3 to 0.6 m/s (1-2ft/s ) Head loss through bar screen .

$H_L=B(W/b)^{4/3} hr \sin\theta$ , $H_L$=head loss .

B=2.42-0.76.

$H_r$=horizontal velocity .

θ =slop of screen .

maximum Head loss (150mm-300mm)

Plain sedimentation :

1-discrete partials :discrete partials (spherical) use

newtons law :-

$v=[\frac{49\,(\rho_S-\rho)d}{3c_D\rho}]^{1/2}$

v=terminal velocity setting (L/T),$L_s$ mass density of the panticale ($M/L^3$) , ρ=Mass density of the fluid ($M/L^3$) , g=acceleration gravity ($L/T^2$), d=diameter of the particle (L), $C_D$=drag coefficient(dimension less )

$C_D = \frac{24}{NR} + \frac{3}{\sqrt[2]{NR}} + 0.34$ ,NR <0.5

NR=reynold number =$\frac{vdL}{M}$ ,μ =absolute viscosity of flute (μ/LT)

$3/\sqrt{NR}$+0.34 Very small so it neglected .

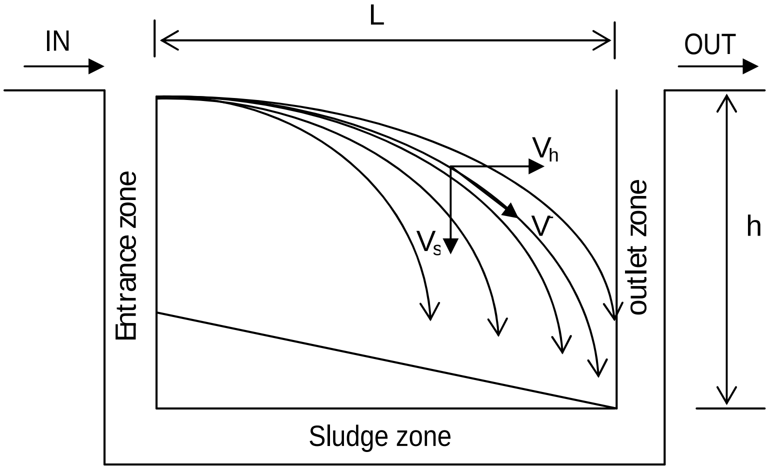
$$\therefore CD = \frac{24}{NR} = \frac{24\mu}{Vd\rho}$$

$V = \frac{g(\rho_S - \rho)d^2}{18M}$ ……..Stokes law .

Design of sedimentation tank on accept of ideal sedimentation particles entering the basin will have horizontal .

Velocity =the velocity of fluid flowing .

$v_h = \frac{Q}{A} = \frac{Q}{Wh}$ and vertical velocity v($v_s$) =terminal setting velocity .




If practical have V‾ it will be removed :

$\frac{V_S}{V_h} = \frac{h}{L}$ , $V_S = \frac{Vh}{L}$ , $\frac{h}{L} \times \frac{Q}{wk} = \frac{Q}{WL} = (SOR)$

(SOR) surface over flow rate = flow / plan area of basin $=\frac{Q}{A}$ physically represents the setting velocity of the slowest setting practical . which 100% removed .if $V_P \leq$SOR it will not be removed , if $V_P$ < SOR removed , SOR (m/day ).

Filter Media :-

The size of filter media is specified by the effective size (E.S.) which is the sieve size in millimeter .that permits 10 percent by weight to pass . uniformity coefficient (U.C. ) : Which is the ratio between the sieve size that will pass 60 percent by weight and the effective size .

Sand : is the cheapest filter medium and has been widely used . sand (5%) by weight after being placed in (40%) HCL for 24 hrs . depth alone (600-700) mm, E.S.( 0.45-0.5) ,c <1.7>1.2 .

Gravel :usually filter media under laind by (400to 600 )mm of gravel which serve to support the sand , permit the filter water to move freely towards the under drains also allows the wash water to move more or less uniformity up ward to sand .

Should be hard , rounded , durable ,weight =1600Kg/$m^3$ be free from flat , thin , or long pieces , no loam ,sand clay ,shell ,or any foreign material .

| | | |
|---|---|---|
| Size :- | 40-60mm | 120-200mm |
| | 20-40mm | 80-120mm |
| | 10- 20mm | |
| | 5-10mm | 60-80mm |
| | 2.5-5mm | 60-80mm |
| | | 400-600mm |

Colloidal particles :as results of their small sizes have Avery target ratio of surface area to volume i-e 1 $cm^2$ material if divided in to cubes of $10^{-5}$ mm on side will have a surface area of 600$m^2$ size of colloidal particles ranging from 0.000001-0.0001mm diameter so due to this immerse area surface a chemical phenomena predominate adsorptions results , with a tendency for other substances to concentrate on the particle , surface panicle charges results from preferential adsorption or from ionization of chemical groups on the surface . most colloidal particles in water and waste water are negatively charge as shown in fig . of the potential energy of interaction for colloidal .

lager and the point of electro neutrality in the solution . this potential can be estimated or measured .

Coagulation : is a chemical teqnique directed toward destabilization of colloidal particles .

History of coagulation was know by English philosopher Sr. Francis bacon at 1620 as one stage of water purification .

As chemical process, common chemical used in coagulation include alum (alum sulfate ) $AL_2(SO)_3$. 14.3 $H_2O$ OR 18 $H_2O$ , Ferrous sulfate $Fe_2(so_4)_3$ sodium aluminates ,ferrous sulfate Fe $SO_4$ and lime $ca(OH)_2$ alum contain about 15-22% of $AL_2O_3$ .

Alum is used (mostly ) for water containing appreciable amoral of organic matter .

If water contain significant alkalinity or natural alkalinity .

*$AL_2(SO_4)_3$-14.3 $H_2O$ + 3CA $(HCO_3)_2$ $\longrightarrow$ 2AL $(OH_3)\downarrow$ +3$CaSO_4$+14.3 $H_2O$ +6 $CO_2$.

*1 mg/l of alma decreases water aka . by 0.5 mg/l as Ca $Co_3$and produce 0.4 mg/l $Co_2$.

If water contains insufficient alkalinity , lime $Ca(OH)_2$or sad ash $Na_2Co_3$ is fed to provide the necessary alkalinity .

$Al_2(SO_4)_3$ . 18 $H_2O$ +3$Ca(OH)_2$ $\longrightarrow$ 2AL (OH)$\downarrow$ +3$CaSO_4$ +18 $H_2O$

$AL_2(SO_4)_3$ .18 $H_2O$ +3$Na_2CO_3$+3$H_2O$ $\longrightarrow$ 2AL $(OH_3)\downarrow$ +3$Na_2$ $So_4$+3$CO_2$+18 $H_2O$ .

Iron wide PH range : effective in removing color , move costly

3Fe $Cl_3$+3$Ca(HCO_3)_2$ $\longrightarrow$ 2$Fe(OH_3)\downarrow$ +3Ca $Cl_2$+6$Co_2$

Or with lime 2$FeCl_3$+3$Ca(OH_2)$ $\longrightarrow$ 2 Fe $(OH)_3\downarrow$ +3$CaCl_2$ .

The optimum PH range for each of the coagulant is :

Alum (4-7 ), Ferric chloride (3.5-6.5 )and above 8.5 ,ferric sulfate (3.7-7)and above 9.0.

M.W: AL =27 ; Fe=55;Cl=35 ; Ca=40 ;O=16 ;S=32;H=1

***Polymers ;*** a polymer molecule is a series of repeating chemical unit hold together by covalent bond .if the repeating units are of the same molecular structure ; the compound is called a home polymer , if the molecule is formed from are than one type of repeating chemical unit is called a copolymer special classes of polymers distinguished from ordinary polymer molecules tyionizable functional groups along the polymer chain are called polyelectrolyte. when these groups dissociates the polymer molecule become charged either positively or negatively .

The charged sites of the dissociated then become available as bonding sites for charged colloid.

Homopolyer anionic :- $-CH_2-\underset{coo^{+}}{\underset{|}{CH}}-CH_2-\underset{coo^{-}}{\underset{|}{CH}}-CH_2-\underset{coo^{-}}{\underset{|}{CH}}-$

polyelectrolyte are excellent coagulants which may be used alone ; or in conjunction with metals coagulants , as alum .

mixing provides for the rapid dispersion or chemical addivity in the raw water and requires a high degree of turbulence and power dissipation . used baffled piping , baffled channel , hydraulic jumper mechanical paddle turbine , propeller .

detention time 10-120 sec ; power required 2-5 kw , per $m^3$/min

(1-2hp/ft/sec )power input expressed in mean velocity gradient G(1/T),$G=(P/\mu v)^{1/2}$,P/V =W

P=power dissipated (FL/T) ,μ=absolute viscosity (μ/LT), V=Volume to which (P)is applied ($L^3$).

GT=30000-60000 ,T=60-120sec .

**<u>Example:-</u>**

flow 25.0 mgd ; Temp =40-60 $F^0$, design rapid mix unit used G =300/sec , T=30sec ; $\mu=0.273*10^{-4}\frac{N.sec}{ft2}$

Solution :- tank volume = 25mgd *$10^6$ $*\frac{gall}{m\ gall}\ (\frac{day}{24*3600})*\frac{ft^3}{7.48gall}*\frac{ft3}{7.48galt}$*30sec =1160$ft^3$

$G=(\frac{W}{M})^{1/2}=300*300*0.273*10^{-4}\frac{N.Sec}{ft2}$

$W=2.46\frac{n/s}{ft2}$

P=w.v∴ $2.46\frac{\mu}{sec.ft^2}*\frac{hp}{550N.ft/s}*1160ft^3=5.14hp.$

Volume $=(D/2)^2\pi H$ if D=H ,$D=(\frac{1160*4}{\pi})^{1/3} \therefore = 11.34\ ft.$

Use two tank D=9ft , H=9ft , hp for each =20.5 hp .

Flocculation: the microflocs formed in rapid mix must boggle me rated to larger size in order to settle under gravity this agglomeration is dependent upon mutual cousins between the micro flock such that they will graw in size .

Flocculation : is also mixing process on which destabilized colloidal particles are brought in to intimate constant in order to promote their agglomeration . the rate of agglomeration or flocculation is dependent upon the number of particles present the relative volume which they occupy and velocity gradient (G) in the basin .

Design factors : detention time 20-30 min , G =25 -65 $sec^{-1}$ use mechanically agitation with rotating paddle wheels or vertically mounted turbines ,see fig . 9-21,22,23.

Design criteria :-

| | Dt(hr) | weir overflow | surface |
|---|---|---|---|
| Sedimentation . | 3-5 | | |
| Presedementation . | 3-8 | | |
| Coagulation & flocculation . | 2-8 | 250 | 20-33 |

* For coagulation or rapid mix:

G:300< G < 1000 $Sec^{-1}$ ,T: 10<T<120 sec

Hp:0.25 < Hp< 3hp /million gallon .

$G=(p/v\mu)^{1/2}$ $G=Sec^{-1}$, $V=m^3$, $ft^3$, p=k w ( N.W/Sec), hp

$M=N.S/m^2$, I b .sec /$ft^2$

*for Flocculation :

G= 25<G < 65 $sec^{-1}$, T=20 <T<30 min , GT=23000-210000

$G=(P/V\mu)^{1/2}$,$G=(\frac{CDA\rho VP^3}{2V\mu})^{1/2}$

$P=\frac{CDA\rho VP^3}{2}$ ,A=paddle area $(L)^2$

A=$m^2$,$ft^2$ <15-25% of area of Basin , $C_D$=drag coeff .

$V_P$=70-75% of speed of paddles =3-0.3 fps

Example :- flow 25 mg C I , flocculation 100 ft long , 50ft wide , 16ft depth , revaluing paddles of 8 in wide , 48 ft long centered 6ft . assume velocity of water 70% of paddle velocity , $C_D$ =1.9 .Find (a)difference in velocity bet . paddle and water (b) G (C)the retention time .

Solution : -

(a)Rotational speed : $V_P=\frac{2\pi rn}{60}$ , $V_P$=paddle velocity Rpm or fps , n=number of revolution per minute , r= distance from shaft to center of paddle ft ,m.

$$V_P=\frac{2\pi 6*1.5}{60}=0.94fps$$

∴the difference in velocity of paddle and water =0.78*0.94=0.66 fps

(b )$G=(\frac{C_D\,A\rho VP^3}{2V\mu})^{1/2}$ OR $p=C_D A\rho\frac{V^3}{2}$

$G_T=(\frac{p}{v\mu})^{1/2}$,$\rho=\frac{\gamma}{g}$

$C_D$=1.9; A of paddle =4*4*48*$\frac{8}{12}$ =512 $ft^2$

$V_p$= 0.66 fps ; P=1.9*512*$(0.66)^3*\left(\frac{62.4}{64.4}\right)=\frac{271(ft-b)}{sec}$/550=271 hp

V. of tank=100*50*16=80000 $ft^3$, μ at 50 F =$2.73*10^{-5} \frac{Ib.sec}{ft^2}$

$G=(\frac{2.71}{80000*2.73*10^{-5}})^{0.5} = 11.1\frac{fps}{ft} or\ sec^{-1}$

(c) (T ) of flocculation tank =$\frac{80000*7.84*24*60}{25*10^6} = 34.5min = \frac{V}{Q}$

GT=11.1*34.5*60=$2.3*10^4$

## Filtration:-

The granular media gravity filter is the most common type used in water treatment to remove non settle able flocks , the production is clear and sparkling water , with the most safety so far as disease is concerned , require the use of filter , two types of filter :-

1- Slaw sand filter : was first used in Britain early in the nineteenth century.
2- Rapid rate filter : also know as the American and mechanical filter , as developed in u.s during period 1900 to 1910 .

Rapid rate filter :

Rapid filtration generally implies process which includes coagulation , flocculation , clarification , filtration and disinfection , see fig . (10-1), (10-2) .

The essential characteristics of rapid filter are :

a- Careful pretreatment of the water in proportion for filtration turbidity of the water applied to the filter preferably < 5 NTU .
b- High rate of filtration 120-240 m/day (2-4 gpm /$m^2$) or more .
c- Washing the filter units by reversing flow of filtered water up ward through the filter to remove mud and other impurities.

Theory of filtration through coarse media :

A rapid filter consist of abed of coarse media such as sand ranging in depth from 300 mm to several meter (<2-3m) the kinetics of particles y short consisting of transport and an attachment step the transport to the surfaced of the filtration media may be produced by diffusion ,interception ,settling , impingement or by hydrodynamic cartilage .

The transport process is aided by flocculation in the interstices of the filter and by the relatively short travel required for removed by sedimentation .

Filtration rate :

For many years the filtration rate used a bout 120 m/day , but the experimental test showed the range (600-1200)m/d is successful .

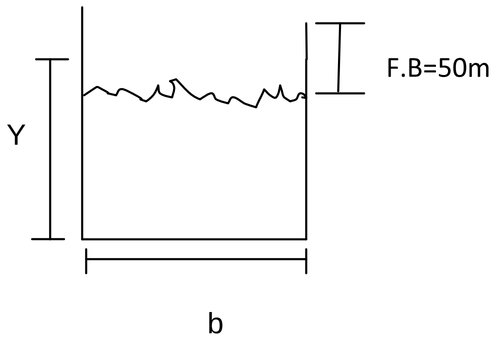
The filter unit and wash water troughs:-

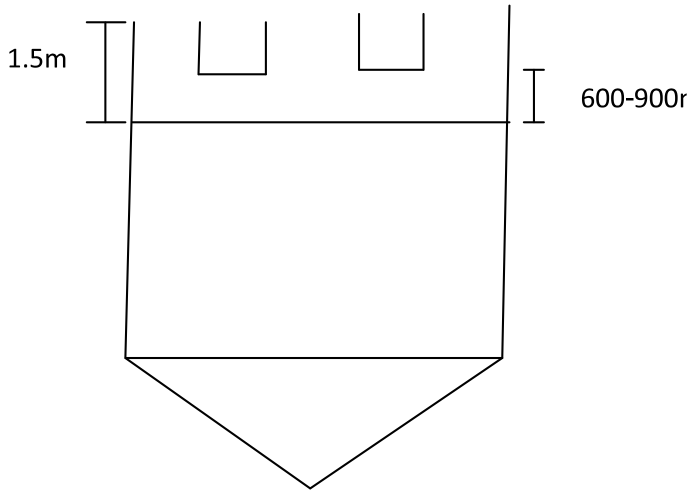
$$y = 1.73\left(\frac{Q^2}{gb^2}\right)^{1/3}$$

Where : Q= discharge , y depth of water

b=width of trough .

g=acceleration .



$S = (\frac{nv}{r^{2/3}})^2$, s= slope of channel ,n=coefficient of roughness ,r=hydraulic radius , v=velocity

Ex : a rectangular trough is to receive wash water from a section of filter bed 2m wide and 3m long the wash water rate is 0.6 m/min , assume y=b .

Solution :- Q=0.6×3×2=3.6m$^3$/sec

=0.06m$^3$/sec

$y = 1.73(\frac{Q^2}{gb^2})^{1/3}$

Y=[(0.06)$^2$/9.81 b$^2$]$^{1/3}$

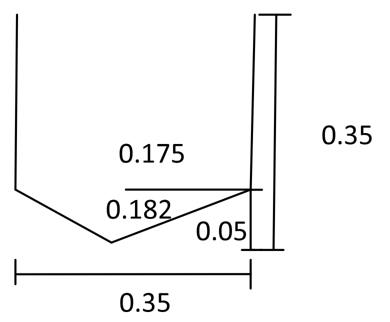
$y = b = o.29m$

A=0.084m$^2$+50 F.B use y=b =0.3 m

V=Q/A=0.06 /0.17*0.3 if critical velocity ,critical depth y/1.73 =0.17

V=1.18

$$s = (\frac{nv}{r^{\frac{2}{3}}})^2 = \left(\frac{0.013 * 1.18}{0.0797^{\frac{2}{3}}}\right)^2 = 0.007m/m$$



**Washing process:-**

1-turbidity break through exceed design limit (>5 NTV )

2-head through bed reach (2-3)m (7-10)ft.

$V_b = V\ t{*}f^{4.5}$

$V_b$= back wash water velocity

$V_t$=terminal velocity , f= porosity of medium

$V_t$=10 $D_{60}$ , $D_{60}$ = u .c *E.S

$D_{60}$= product of the effective size and uniformity coefficient in (mm)

The amount of water for washing from (1-5)% of that product . continue for (5mm)back wash rate >0.3m/min see example page .253

Air wash system :-

Use air from (0.3-1.5 ) m/min(7gallon /min /$ft^2$)for few minutes followed by water (0.3-0.5)m/min (7-12gall/min /day )for (10-15min)

Disinfection :-is the destruction of harmful microscopic organisms (mainly bacteria )in the water .

apply chemical disinfectant to water chlorine destroy for organisms in two

Step :-

1-chlorine molecules penetrate the cell wall of the harmful organism .

2-the organism enzymes react with the organisms is destroyed .

Three forms of chlorine are used :

1-chlorine ($CL_2$)liquid or gas

2-calicium hypochlorite $Ca(OCl)_2$

3-sodium hypochlorite Naocl

Chlorine in liquid from is amber –colored , 1.5 times as eleuse as water supplied to treatment plant in pressurized container , liquid chlorine changes to chlorine gas at ream temperature and press ore the gas is (2.5 )times as dense as air is greenish yellow color .

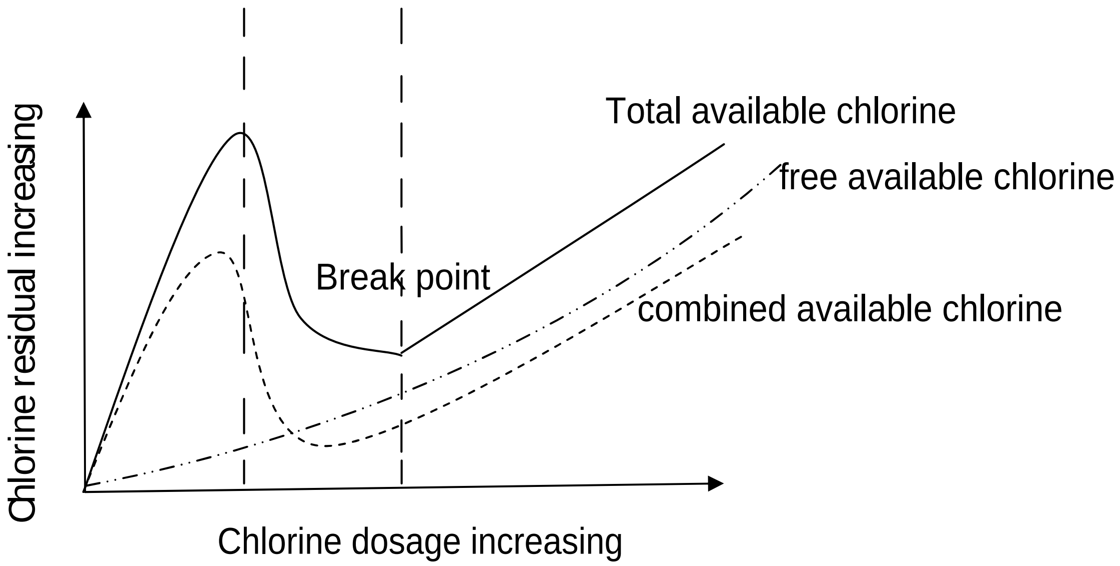
Breathing in a few breaths of chlorine at concentration 1000 ppm (0.1%)

$$CL_2 + H_2O \rightleftharpoons HOCL + HCL$$

HOCL is very weak acid but strong disinfection depending PH water valne HOCL can disassociate (break up in to ions)

$$HOCL \rightleftharpoons H^+ + OCL$$

The free available residual at the consumer shovels at least 0.2 mg/L ,this level helps to ensure that the water is free from harmful bacteria



# (SEWERAGE)

**Definitions:-**

**Sewage :** is the liquid conveyed by a sewer , it may consist of any one or a mixture of liquid wastes .

**Sanitary sewage or domestic sewage :** is that which originates in the sanitary conveniences of a dwelling , business building , factory , or institution .

**Storm sewage :** is liquid flowing in sewers during or following a period of rain full and resulting there from .

**Infiltration :** is the water that has leaked in to sewers from the ground.

**Inflow :** is water which enter sewers from surface sources such as cracks in manholes , open clean outs , perforate manholes cover , and roof drains or basement sumps connected to the sewers .

**Sewer :-** is a pipe or conduit , generally closed but normally not flowing

Sanitary sewer , storm sewer ,and combined sewer .

**Combined sewer :** is designed to carry do mastic sewage , industrial waste and storm sewage . a sewer composed of combined sewer is known as a combined system , but if storm sewage is carried separately from the do mastic and industrial waste is called a separate system .

**Sewerage :** is applied to the act of collecting , treating and disposing of sewage .

**Sewage treatment :** covers any process to which sewage is subjected in order to remove or alter its subjection able constituents so as to render

**Sewage disposal :** is the act of disposing of sewage any method . it may be done with or with or without previous treatment of the sewage .

## Characteristics of sewage

### Physical characteristics:

Sewage is over 99.9% water , but the remaining material has very

significant effects .

**Solids determinations:** the solid in sewage may be suspended or insolution .

**Total solid** include both suspended and dissolved is determined by evaporating at (103-105℃ )of a known volume or weight of sample and weight the residue , mg/L .

**Suspended solid :** filter (use filter size 1μm ) a know volume or weight of sample , dry both , the change in weight (of total dry solid and dry paper and pre weight dry paper ) dividing the sample volume or weight yields the suspended solid concentration .

**Dissolved solid :** a measured volume weight of filtrate is evaporated to dryness and the residue weighted to determine the dissolved solid(use 1μm paper)

**Volatile solids:** are those solids ignitable to 550 ℃ and it is a rough measure of organic content or content ration of biological solids such as

**Total volatile solid :** ignition of the residue from , the total solid test in

**Volatile dissolved solids:** ignition of dry dissolved solid at 550℃ weight divide by sample volume . mg/L

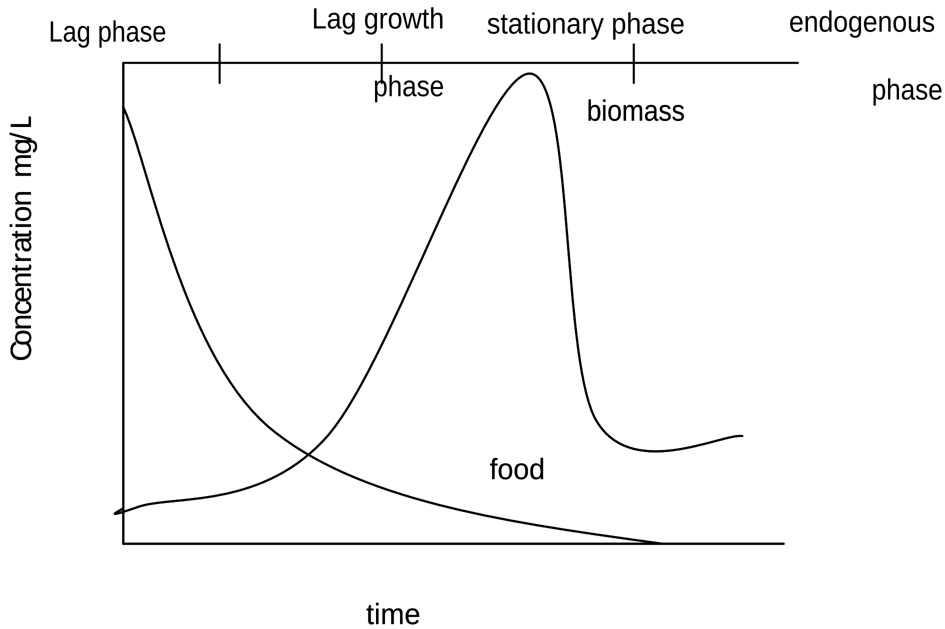
volatile dissolved solid or difference of residue following drying and the following ignition .

**Microbiology of sewage and sewage treatment:**

waste water contains bacteria ranging 500,000 /m L to 5,000,000 /m L waste water also contain viruses , protozoa , worms ,etc .

**Bacteria:** are single called (plants ) which metabolize soluble food and reproduce by binary fission . bacteria are capable to solubilizing food particles outside the cell by means of extra cellular enzymes and hence can remove soluble, colloid , and solid organics matter from waste water

**Growth and food utilization:** the relation ship of cell growth and food utilization can be illustrated by simple biomass curve .




**Biomass Growth and food utilization**

**Lag phase** :in this part the micro organisms must first become acclimated to their surrounding environment and the food provided there . once growth has been initiated , it will proceed rapidly , bacteria reproduce by binary fission . maximum growth accurse at logarithmic rate phase called log growth phase . food supply become limited , the cell will not be able to obtain food from external source endogenous catabolism of stored protoplasm far ma other cells maintenance energy other cells die and lyses or break open and releasing their protoplasm then biomass slowly decreases .

**Stationary phase :** represent the time during which the production of new cellular material is roughly offset by death and endogenous respiration .

**Aerobic processes :** aerobic bacteria utilize free oxygen as an electron acceptor , end product are $CO_2$,$H_2O$ ,$SO_4$, $NO_3$, $NH_3$, more bacteria and heat .

An aerobic process : an aerobic bacteria oxidize organic matter utilize produce $CO_2$,$H_2O$,$H_2S$ ,$CH_4$,$NH_3$,$N_2$,reduced organics , more bacteria and heat .

Chemical characteristics: waste water contain organic and in organic chemicals ,

Organic : fats , portions , carbohydrates , acids, etc.

In organic : Nitragen , phosphorrus , (may be organic) and alkalinity .

Biochemical oxygen demand (BOD): the amount of oxygen consumed during microbial utilijaticer of oxygen .

The exertion of BOD is considered to be a first order reaction dy/dt =$k_1$y, y is the BOD remaining at time t and $k_1$ is constant . by integration , let L=BOD at t=0

$y = Le^{-k_1 t} \therefore BOD$ exerted at time t is =L-Y

$= L(1 - e^{-k_1 t})$

L: initial percent , y:remaining .

BOD $_5$,5 represent oxygen demand in 5 days of lest the value of $k_1$and L determined by a series of BOD test . Thomas developed the aprox. Formula:

$BOD = Lk_1 t(1 + \frac{k_1 t}{6})^{-3}$

$$\therefore (\frac{t}{BOD})^{1/3} = (k_1 L)^{-1/3} - \frac{K_1^{2/3}}{6L^{1/3}} t$$

Plot $(\frac{t}{BOD})^{1/3}$ V st a straight line obtained $(k_1 L)^{-1/3}$ intercept at t=0 , and slope $k_1^{2/3}/6L^{1/3}$ , $K_1$ varies with temp .

$K_1$(T)=$K_1$(20) [1.047],and slope $K_1$ (20):at 20℃ (T): actual temp.

Streeter –Phelps equation :

Do inflow + do source – (Do outflow + Do skins )

=charge in DO storage in segment .

Q(DOS) +g(AR)V-[Q {(DOS)+$\frac{d(DOS)}{dx}$ Δx ]+f ($BOD_w$+$BOD_s$)V}=$\frac{V(DOS)}{\Delta t}$

V=volume of water contained in the stream from Btoc. g(AR) is function of reaction and f($BOD_w$ +$BOD_s$ ) is function of exerted by the waste in the stream segment per unit time . Q=VA(V velocity , Across sedan area )

Divided by v and $\frac{\Delta(DOS)}{\Delta t}$ = -V$\frac{\partial(DOS)}{\partial x}$ -f($BOD_w$+ $BOD_s$) +g(AR)

$\frac{\Delta(DOS)}{\Delta t}$=0 if DO not change with time .

V$\frac{\partial(DOS)}{\partial x} = f(BOD_w$+$BOD_s$)+g(AR)

By arrangement and integration streeter – Phelps will be :

$$d = \frac{k_1 L_0}{K_2 - K_1}(e^{-\frac{k_1 x}{v}} - e^{\frac{-k_2 x}{v}} + D_0\, e^{-k_2 x/v}$$

Or $D = \frac{k_1\ L_0}{k_2 - k_1}(e^{-k_1 t} - e^{-k_2 t}) + D_0 e^{-k_2 t}, \frac{x}{v} = t$

$L_0$ = initial BOD exerted on the stream by waste +BOD and organic in the stream it seff .

$D_0$ =initial dissolved oxygen .

D= dissolved oxygen X units from the initial point .

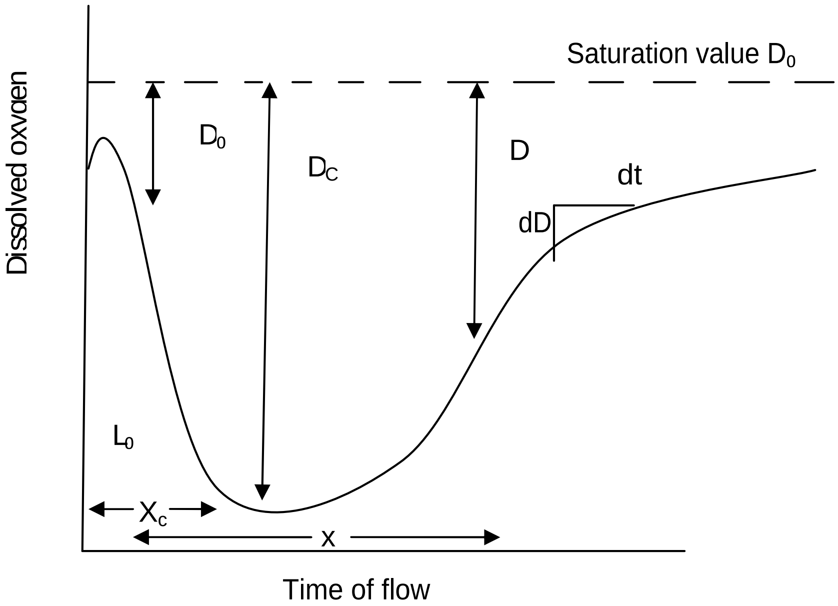
$k_{2=} \frac{(D_l V)^{1/2}}{H^{3/2}}$

$k_2$ = reaeration coefficient (base e ) per hour

$D_L$= diffusivity of oxygen in water =0.000081 ft/hr @20℃

V=velocity of flow ft/hr .

H=depth of flow in ft .,$K_1$=de oxygen coff. /hr, day .




t $_c$=critical time

$$T_C = \frac{1}{K_2-K_1}\ln[\frac{K_2}{K_1}(1-D_0\frac{K_2-K_1}{K_1 L_0})]$$

For more information about streeter - phelps see teat book page

440 -443.

$$k_{2T} = K_2@20^0 \times 1.047^{T-20}, K_{1T} = K_1@20^0 \times 1.047^{T-20}$$

T= temperature .

**Example:** city population 20000 cap. , 120 gpd, S.T.P. effluent BOD = 28 mg/ℓ @ temp. 25.5 ℃ with 1.8 mg/ℓ $D_0$ . stream flow 250 cfs @ 1.2fps depth 8 ft , temp. of stream 24℃ , BOD $_5$=3.6 mg/ℓ before mix with sewage $k_1$=0.5 /d @20℃ . find a)sewage flow cfs , b)$D_0$of mixture of sewage and stream c) temp. of mixture , d)critical $D_0$dificant of river just below S.T.P. e) distance with min $D_0$.

**Solution :-** a) (120 gpd )×(2×$10^5$)=240×$10^5$ gpd = 24 mgpd *1 mgpd =1.547cfs

∴ 24×1.547 cfs =37.1cfs

b)assume $D_0$@24 ℃ =8.5 mg /ℓ

∴ 90% = 8.5 mg /ℓ ×0.9 =7.65 mg/ℓ

∴ $DO_m = \frac{Qr(DO_r)+Q_s\,(DO_s)}{Q_r+Q_s}$

$= \frac{(250\ cfs\ )7.65\frac{mg}{\ell}+(37.1\ cfs\ \times)1.8mg/\ell}{(250+37.1)cfs}$= 6.89 mg/ℓ

c) $T_m = \frac{250\ cfs\ \times 24℃+37.1 cfs\ \times 25.5\ ℃}{(250+37.1\ )cfs} = 24.2℃$

d) initial $D_0$ diffident $D_0$@20 ℃ for stream ≈85 mg/ℓ

∴ dificit =8.48-6.8 =1.59mg/ℓ

e-f) $k_2@20 = \frac{(D_L V)^1/2}{H^{3/2}} = \frac{(81\times\frac{10^{-6}ft}{hr}\times\frac{1.2ft}{s}\times\frac{3600s}{hr})^{1/2}24hr/d}{(8ft)^{3/2}}$

∴ $K_2$ @20=0.627/ day.

$K_2T = K_2@20 \times 1.047^{T-20}$

$\therefore K_2@24.2 = 0.627 \times 1.047^{24.2-20} = \frac{0.76}{day}$

$\therefore K_1@24.2 = 0.5 \times 1.047^{4.2} = \frac{0.607}{day}$

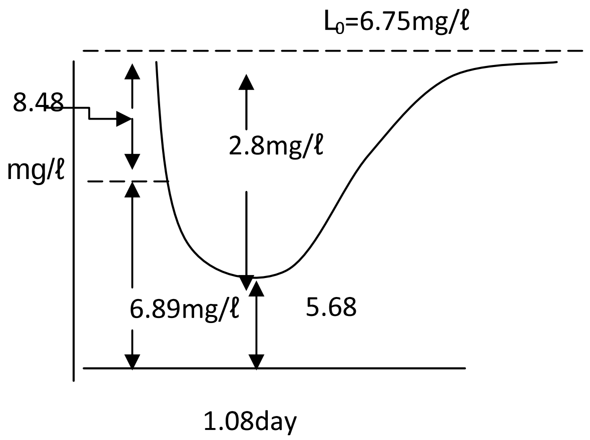
$$L_0 far\ mixture = \frac{250cfs \times \frac{3.6mg}{\ell of BOD} + 37.1cfs \times 28mg/\ell BOD}{287.1cfs}$$

=6.75mg/ℓ of BOD.

$$t_c = \frac{1}{K_2 - K_1} \ln\{\frac{K_2}{K_1}[1 - \frac{D_0\,(K_2 - K_1)}{K_1 L_0}]\} = 1.08\ days$$

$$\therefore D_c = \frac{0.607 \times 6.75}{0.76 - 0.607}(e^{-0.607 \times 1.8} - e^{-0.76 \times 1. \times 08}) + 1.59e^{-0.76 \times 1.08} = 2.8mg/\ell$$

$\therefore X_c$ for min $D_0$=1.2ft/s ×3600s/hr ×24hr/d×1.08day=112000ft.




Example : a waste water effluent of 560ℓ/s with a BOD= 50 mg/ℓ , DO=3 mg/ℓ and temperature of 23 ℃ enter a river where the flow is 2.8m$^3$/s with a BOD of 40 mg/ℓ , DO 8.2 mg/ℓ , and temperature of 17 ℃ , use $K_1$ 0.1 / day at 20 ℃ , velocity of steam 0.18 m/s , calculate the minimum DO and draw oxygen sag curve .

Total flow (stream& W.W .T.P.)=2.8+0.56 =3.36 m$^3$/S

Temp . of the mixture = $\frac{\frac{2.8m^3}{S} \times 17℃ + 0.56 \times 23℃}{\frac{3.36m^3}{sec}.} = 18℃$

BOD=$\frac{\frac{2.8m^3}{S}\times\frac{4.0mg}{\ell}+0.56m^3/S\times 50mg/\ell}{3.36m^3/sec}=11.7mg/\ell$

DO=$\frac{2.8\times\frac{8.2mg}{\ell}+0.56\times 3.0mg/\ell}{3.36}\ 7.3mg/\ell$

$k_1$ at 20°C=0.1/day ∴$k_1$(18)°C=0.1×$1.047^{18.20}$=0.09 /d

$K_2$ at 20 °C=0.31 /day ∴ $k_2$(18)°C=0.3 /day .

$L_0=\frac{5\ day\ BOD}{1-e^{-k_1t}}=\frac{11.7}{(1-e^{-0.1\times 5})}\approx 17.1\ mg/\ell$

$D_0$saturation at 18°C=9.5mg/ℓ

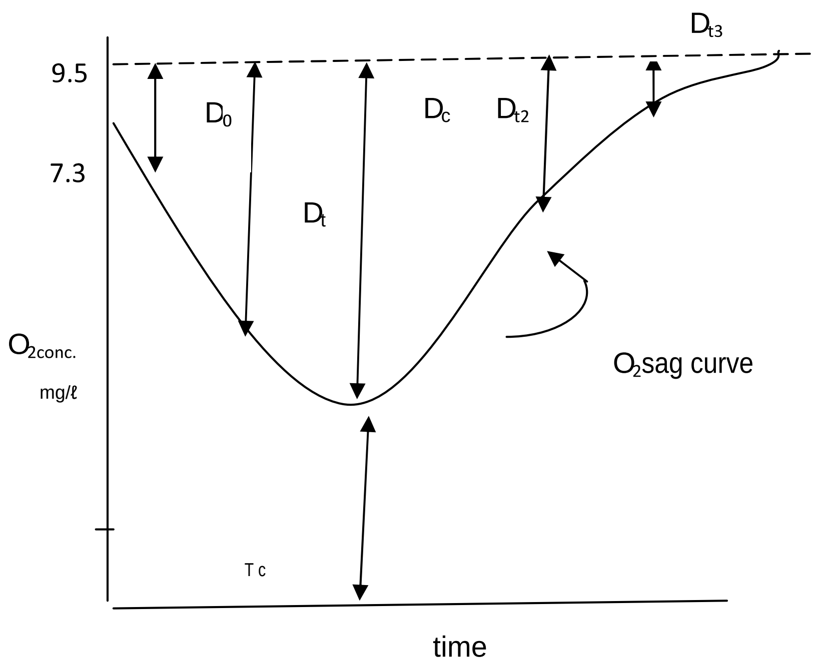
Deficit $D_0$ =9.5 -7.3=2.2 mg/ℓ

$$\therefore D_t=\frac{0.09\times 17.1}{0.3-0.09}(e^{-0.09t}-e^{-0.3t})+2.2e^{-0.3xt}=3.56mg/\ell$$

$$t_c=\frac{1}{0.3-0.09}\ln[\frac{0.3}{0.09}(1-2.2\frac{0.3-0.09}{0.09\times 17.1})]=4.03day$$

Distance of critical conc. =$\frac{4.03day\times\frac{0.18m}{s}\times 24\times 3600}{1000m/km}=26k$m

Minimum $D_0$ is =(9.5-$D_{tc}$)=5.94 mg/ℓ



## Measurement of waste water flow :

_Venture – type flume .

_ par shall flume .

$$Q = 2.23W\ h_a^{1.522W^{0.026}} cfs$$

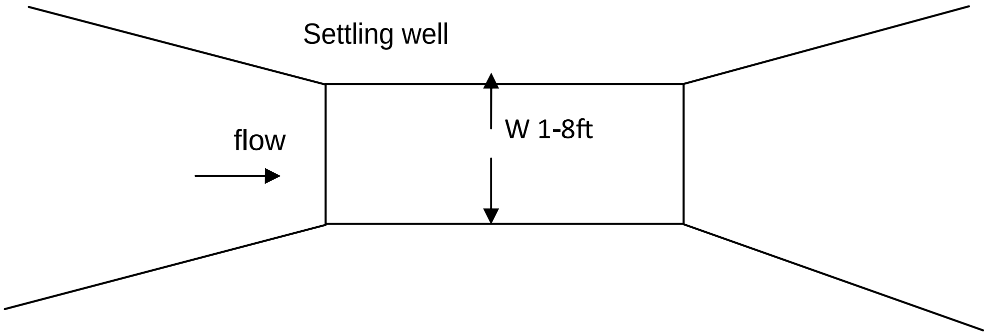
Or

$$Q = 2.23 \times 10^{-2} W \left(\frac{ha}{304.8}\right)^{1.522\left(\frac{W}{304.8}\right)^{0.026}} \quad \left(\frac{m^3}{min}\right)$$

*Q=the flow in c f s , $m^3/min$ .

*W=the width of the throat ft , m.

*$h_a$=the up stream depth in ft , millimeters .




Equation in metric system applied to flumes 300 mm to 3m , 1 to 8 ft the th roat width ,

Example : find the flow through apar shall flume with threat width 2 ft at the max . free . flow head 2.5 ft .

$Q = 4\ Wh_a^{1.522w^{0.026}}$= 4×2×$2.5^{1.522\times 2^{0.026}}$=33.4c f s=21.6 mgd

## Grit Removal :

Grit removal devices rely upon the difference in specific grauity between organic and inoranic solid to effect their separation .all particles are assumed to settle in occord with newtons law .

$$V_s = [\frac{49(\rho_s - \rho)d}{3c_D\rho}]^{1/2}$$

And to be scoured at velocity

$$V_h = [\frac{8\beta(s-1)gd}{f}]^{1/2}$$

When :

*$V_s$=terminal settling velocity (L/T)

*$\rho$ =mass density of the the fluid ($m/L^3$)8

*d=diameter of the particles (L)

*$V_h$=horizonal velocity of flow (L/T)

B =dimension less Darcy –weisb ach factor usually 0.02 to 0.03

To assure removal of grit , while permitting such organic matter as might settle to be resus pended by scour , the necessary conditions may be calculated as Follows :

For particles of grit with adiumeter of 0.2 mm and specific grauity =2.65

$$V_s = [\frac{4(980)(2.65-1)(0.02)}{3\times10}]^{1/2} = 2.1cm/s$$

$V_h$=23 cm/s.

The scaur velocityof organic particles with aspecific grauityof 1.10 and

$$V_h = [8(0.06)(1.10-1)(980)(0.02)/0.03]^{1/2}$$

The basin designed to have asurface over flow rate of 0.021 m/s and harizontal velocity greater than 0.056 m/s and less than 0.23 m/s.

If will remove grit without removing organic matter or material .

The horizontal velocity in grit chambers is usually governed by either of two contoral section for a rectangular contral section such as par shall flume , the channel crass section mast be parablic with

$Y=C\,X^2$

**Example** : Design agrit chamber for a horizontal velosity 250 mm/s , min.flow 20,000$m^3$/day, max. 80,000$m^3$/day , average flow is 50,000$m^3$/day , use 4 mechanically cleaned channels :

Peak flow =2.13×50,000=106650$m^3$/day .

If 4 channels each channel carry

Max =4×5000=20,000$m^3$/day

Min =20,000÷4=5000$m^3$/day

Average =50,000÷4=12500$m^3$/day

For aparabolic channels A =2/3WD

$$Qmax, A = \frac{Q}{V} = \frac{\frac{26.666m^3}{d} \div 60 \times 24}{\frac{250mm}{s} \div \frac{1000mm}{m}} = 1.23m^2$$

$Q_{max}$=20,000$m^3$/d ⟶ A=0.98 $m^2$

$Q_{ave.}$=12500$m^3$/d ⟶ A=0.58$m^2$

$Q_{min}$=500$m^3$/d ⟶ A=0.23$m^2$

Use 1.5 m width of channel .

$D_{max}$=3A/2W

$D_{max}$=3/2(0.93/1.5)=0.93

*$h=v^2/2g+ D_{max}$

For critical depth $dc = V_c^2/g$

If head less in the cantral section is 10% of velocity head .

Bernoullis equation may be written as :

$$D = \frac{V_c^2}{g} + \frac{V^2}{2g} + 0.1V_c^2/2g$$

$$D = 3.1V_c^2/2g$$

$$V_c = (\frac{D2g}{3.1})^{1/2}$$

$$dc = \frac{V_c^2}{g} = \frac{(2.42)^2}{9.8} = 0.60m$$

The area of the contral section is

$$Wd_c = \frac{Q}{V_c}$$

$$W = \frac{Q}{Vcdc} = \frac{\frac{20{,}000m^3}{d} \div 60\times60\times24}{\frac{2.24m}{s}\times0.6m} = 0.16m$$

Or the other flow condition :

$d_c = (\frac{Q^2}{W^2g})^{1/3}$ when $Wd_c = \frac{Q}{V_c}$ $\&d_c = V_c^2/g$

And from berralls eguation .

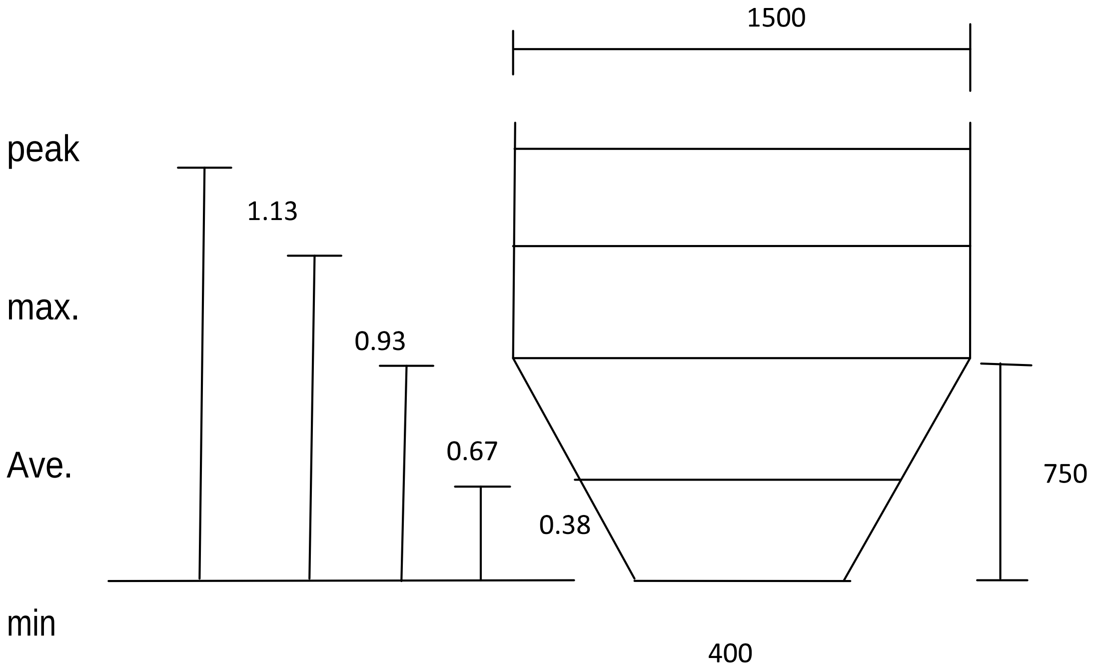
D=(3.1/2)$d_c$ when $D = 3.1V_c^2 2g\&Wd_c Q/V_c$

For the parabolic section :

W=(3/2)(A/D)

Result tabulated as below ang fig :

| Q | $d_c$ | $W_c$ | D | W |
|---|---|---|---|---|
| 5000 | 0.24 | 0.16 | 0.38 | 0.91 |
| 12500 | 0.43 | 0.16 | 0.67 | 1.30 |
| 20,000 | 0.60 | 0.16 | 0.93 | 1.50 |
| 26,666 | 0.73 | 0.16 | 1.13 | 1.63 |




parabolic grit chamber for use with rectangular contreal secttion .

length of grit chamber depends upon trajectory of the slowest setting flow V settling 21 mm/s , harizontal velocity 250 mm/s

$V_h/V_s$=L/D ,L=D($V_h/V_s$)

*see the fig

L=(250/21)D≈ 12D use D=1.13 m ,L=13.5m

## Primary Treatment system :-

**Plain Sedimentation :**the theory of sedimentation of discreat , flocculent and hinderd suspensian which studied at first can be used for this part . chp.9

**Primary clanifires :** removal rang (30-60%)with in settling rates of

(0.3-0.7 ) mm/s . retention time 1to 2 hrs at peak flow . over flow rate of (1 to2.5 )$m^3/m^2$/hr .depth of (1to 5 )m (3to16ft ) use (2to5)mand use at least two basins.

Fig . 23-1, fig. 23-2 can be used for design.page . 470 steel.

**Example:-**Design primary clarification system for ave. Wast water flow 7500 $m^3$/day .max 18000 $m^9$/d . min . 4000$m^3$/d.

Use at least two basin to remove 35% of BOD at peak flow when all basin in operation and 30% at peak with n-1 basin .find the efficiency at the other flow condition ?

**Solution :**from fig 23.2 for 35% at the surface over flow rate is 26.5 m/d .

Total area reguired for all basin is :

$A_n$=18000/26.5=679m^2,A_(n-1 )=18000/51=353m^2

$n \geq 2$ use n>2 ,n=3

3 basin ,each 226 $m^2$ or 2 basin ,each 353$m^2$

$$\text{Depth} = \frac{\text{flow } \times \text{retention time}}{\text{Area}} = \frac{18000\text{m}^3/\text{d} \div 24\text{hr}/\text{d} \times 1\text{hr}}{353}$$

Depth =2.1 m use .

When retention time 1 hr and one basin is used .

| Flow | NO-basin | Area | SOR Q/A | Efficiency |
|---|---|---|---|---|
| 18000 | 2 | 706 | 25.5 | 35.1 |
| 18000 | 1 | 353 | 50.0 | 30.0 |
| 7500 | 2 | 706 | 10.6 | >37.0 |
| 7500 | 1 | 353 | 21.3 | 36.0 |
| 4000 | 2 | 706 | 5.7 | >37.0 |
| 4000 | 1 | 353 | 11.3 | >37.0 |

The two $353m^2$basins whould have over flow rat and efficiency is shasen

**Some design condition of :**

Rectanguler tank:length /width 4/1

Minimum depth 2 to 6m average 3.5m (12 ft )

Weir loading rate =120to 370 $m^3$/d per meter of weir length (10000to 30000gal/ft per day )

Velocity of sludge and scum :1m/min (3ft/min )

Circular tank =depth 2to4m (6to13ft)

Weir loading rate :

Sludge and scum =1.5 to 2.5 m/min (5 to 8ft )minim.

## Secondary Treatment system:

**Attached Grawth processes or suspended grawth processes :**

it utilize a solid meduim upon which bacterial solids are accumlated in order to main tain a high population .

The area a vailable or or such grawth is an important design parameter .

1- **Intermittent sand filter :-see (pag 479 steel )**
2- **Trickling Filter :-**

Grawth medium such as rock or formed plastic shape bacterial grawth medium occurs upon the surface while oxygen is provided by air diffusion through the void space.fig (24-3) Nutrient and oxygen are transferred to the fixed water layer and waste products are transferred the moving layer, primarily by diffusion with growth depth increase the layer slim .

The solid in the filter effluent are removed from the flow in a secondary clarifire.

**Filter class fication :-** the filter is described by hydraulic or organic loading rate or by the medium prouided for bacterial grawth.

**Organic loading rate** :- 0.3 to 1.5 Kg $BOD_5/m^3$/day for built filter volume.

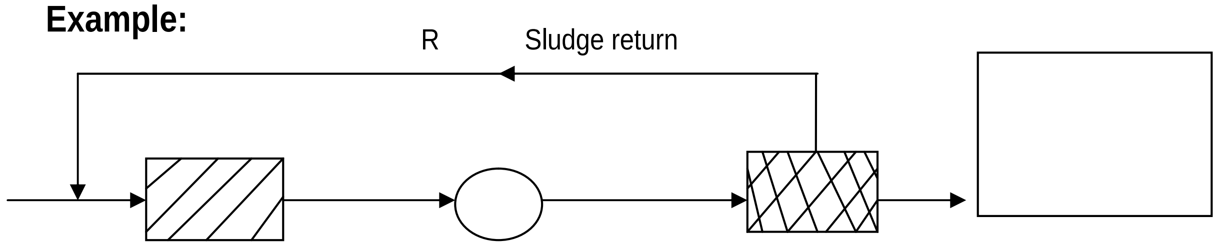
**Hydraulic loading rate :-** 1870 to 3740 mm/day

Media :erushed stone, sand,gravel,60-90mm.

**Recirculation :**- see fig 24-6

At least fourteen different can figuration in use .

**Example:**




see other configurations.

Recirculation rates ranges from 50 to 1000%usually 50-300 percent.

(300%) that means through 12 hr. the water recgcle 3-time.

**Efficiency of Trickling :-**

$\frac{C_i-C_e}{C_i}=\frac{1}{1+0.532\sqrt{\frac{QC_i}{VF}}}$ for both single or two stage system.

$C_i$= influent $BOD_5$ mg/ℓ

$C_e$= effluent $BOD_5$ mg/ℓ

V= filter volume $m^3$

F=recirculation factor given by :

$F=\frac{1+r}{(1+0.1r)^2}$ r= is the ratio of the recirculation flow to the raw waste

For second stage :

$$\frac{C_e-C_e'}{C_e}=\frac{1}{1+\frac{0.532}{1-[(C_i-C_e)/C_i}\sqrt{\frac{QC_e}{V'F'}}}$$

$C_e'$= effluent BOD from 2nd stage.

$V'$= Volume for 2nd stage.

$F'$=reciculation rate ractor for 2nd stage.

**Example:-**Design of trikling filter system single – stage with 2m depth,effluent $BOD_5$ of 30 mg/ℓ,influent $BOD_5$ =160mg/ℓ flow = $10^4 m^3$/day and hydravlic flow rate or hydravlic loading rate 20m/day.

Solution :-from the formula

$$\frac{160-30}{160} = \frac{1}{1+0.532\sqrt{\frac{6.94(160)}{VF}}}$$

VF=5901 if depth =2m ,V=2A ,A=area

$$Area = \frac{flow}{Hydravlic\ loading\ rate} = \frac{10^4(1+r)m^3/d}{20}$$

VF=5901 $F = \frac{1+r}{(1+0.1r)^2}$ ,2AF=5901

F=2.08, V=2837$m^3$ use A=1500$m^2$

Check $C_e$ to above conditions ,V=3000$m^3$

$$\frac{160-C_e}{160} = \frac{1}{1+0.532\sqrt{\frac{6.94(160)}{6000}}}, C_e = 29.8mg/\ell$$

Filter depth 2m, A=1500$m^2$,V=2837$m^3$,recirculation ratio 2, reciculation factor F=2.08 eff.$BOD_5$29.8mg/ℓ inff. $BOD_5$ =160 mg/ℓ & flow rate $10^4 m^3$/d,eff.=81.4%.

**Example:-** find the $BOD_5$ two stage trickling fliter when :Q=3.15$m^3$/min,$BOD_5$ =290 mg/ℓ,volum of filter No.2 =830 $m^3$ fliter depth =2m, recirculation NO.1 =125% Q,reciculation (filter NO.2)=100%.

**Solution:-**first stage from formula :

$$\frac{C_i - C_e}{C_i} = \frac{1}{1+0.532\sqrt{\frac{QC_i}{VF}}} \ \& \ F = \frac{1+r_1}{(1+0.1r_1)^2}$$

$$F = \frac{1+1.25}{(1.125)^2} = 1.78$$

$$\frac{290-C_e}{290} = \frac{1}{1+0.532\sqrt{\frac{3.15(290)}{830(1.78)}}}$$ ,$C_e$=85.5mg/ℓ

For second stage $F' = \frac{1+r_2}{(1+0.1r_2)^2} = \frac{1+1}{(1+0.1\times 1)^2}$=1.6

$$\frac{85.5 - C_e'}{85.5} = \frac{1}{1 + \frac{0.532}{1-[(290-85.5)/290]}}\sqrt{\frac{3.15(85.5)}{830 * 1.65}}$$

- See text book for other formulas to finding the design of trickling filter.
- **NRC.formula:** includs theeftect of secondary clarifires max. useful

  $C_e = (\frac{C_i + rC_e}{1+r})e^{-KD}$ K=0.49 light rate to 0.57 law rate

  D=depth of filter.(m)

  For thepast example

  $$C_e = \left(\frac{290+1.25C_e}{1+1.25}\right)e^{-0.49(2)} = 61.1mg/\ell$$

  $$C_e' = \left(\frac{61.1+1.00C_e'}{1+1.00}\right)e^{-0.49(2)} = 14.1 \ mg/\ell$$

Distribution systemeither ratany nozzles with speed of ratation at least (6rphr ). Or fixed nozzles for continous flow.

**<u>Head loss :-</u>**

Total head loss = enterance loss +friction loss (in pipe ) +distribution (300mm)+elevation of distribution.

Under drain shown in (24-10steel) with velocity enflow (0.6-0.9)m/s usually ventilation in under drain with aper area in Cop of filter block is 15% (at least ), SOR=25-33 m/d.average flow 50m/d peak, solid pratuction 0.2 to 0.5 Kg VSS /Kg BOD removed.,moisture contant 90%-98.

**Reactors and other Hydraulic characteristics:-**

Types of reactors:

1-the batchreactor

2-the plug – flow reactor or flage reactor

3-the continuos –flow stirred –tank reactor –complex –mixreactor

4-The arbitrary –flow reactor

5- the packed –bed reactor.

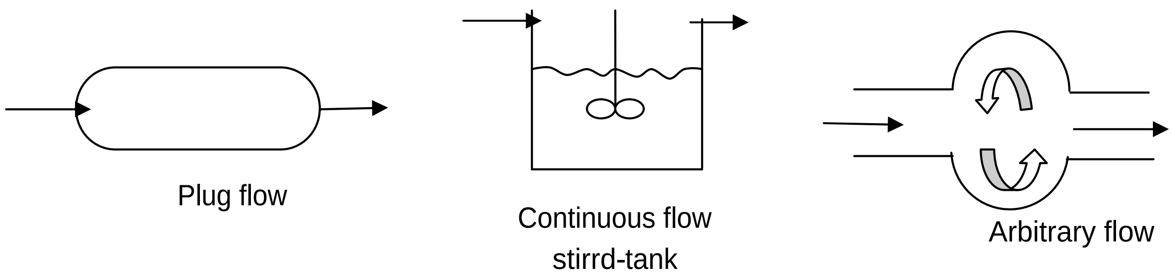
6-Fluidized –bedre.

See fig 5-2 p.1.55.

From 1-4 are hydraulic characteristics….Homogeneous reations.

Usually carried out in such reactors … Hetragenous reations.

**Hydraulic characteristics Reactars.**



Pluge flow – continuous flow of dye fracer to produce concentration c, t=actual time, to= theoretrcal ultimate time V/Q=T, for ideal pluge flow $t=t_0$ at cont. fracer.

| Rate of accumulation of fracer with in the reactor | = | rate of flow of fracer in to the reactor | _ | Rata of flow fracer out the reactor. |
| --- | --- | --- | --- | --- |

Accumulation =inflow – out flow.

$$V\frac{dc}{dt} = QC_0 - Qc \ or \ \frac{dc}{dt} = \frac{Q}{V}(C_0 - C)$$

$C = C_0(1+e^{-t}) = C_0(1-e^{-t/t0})$,

$C=C_0(1-e^{-\theta}),\theta=\frac{t}{t_0}$

From 0 to c & o to t

**Suspended Growth process :-**

**Activated sludge process :**

Principles of suspended Growth process ::

The basic factor in design, control and operation of suspended growth system is mean cell residence time or sludge age $(\theta_c)$

$(\theta_c = \frac{x}{(\Delta x/\Delta t)}$

X=total microbial mass in reactor.

$\Delta x/\Delta t$ =total quantity of solids with drown daily including waste solid.

$\theta_c^m < \theta_c < \theta_c^d$ , $\theta_c^m$: lowest value of $\theta_c$ :lowest value of $\theta_c$ in which operatic is possible.

If $\theta_c < \theta_c^m$ means organisms are removed more quickly than they are synthesized.

$\theta_c^d / \theta_c^m$ safety factor of the system up=20 Table (24-3)

At equilibrium system the quantity of solid produced must equal that lost. The quantity produced per day :

$$M = \frac{\mu^{\sim} S}{k_S + S} - K_d$$

M=net specific growth rate or growth per unit mass per unit time.

$\mu^{\sim}$ =maximum rate of growth.

S=concentration of substrate surrounding the micro organisms.

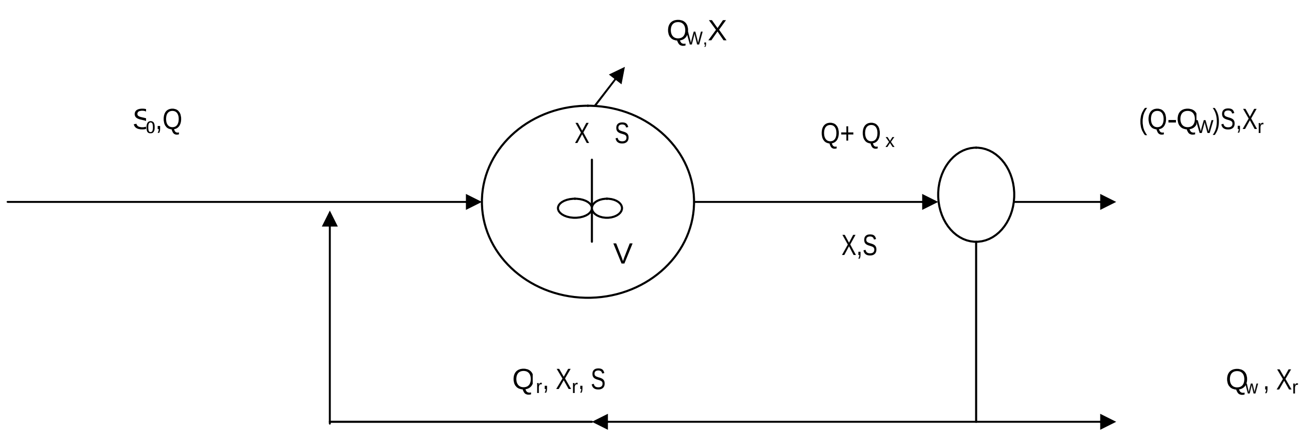
$K_s$= half velocity constants equal to subs ant conc. At which the rate of substrate removal is on half the max. rate.

$K_d$ =micro organisms decay coefficient (mass /unit mass /unit time)reflecting the endogenous born –up of cell mass.

**<u>Completely mixed process with solids Recycle :-</u>**

Can be used to activated sludge systems, the mean cell residence time or sludge age:

$$\theta_c = \frac{x}{\Delta x/\Delta t} = \frac{XV}{Q_w X_r + (Q - Q_w) X_e}$$



Q= the waste flow rate .

X= the mixed liquor suspended solids concentrations ($m/L^3$)

$X_r$=the clarifier under flow suspended solid con cent ration

$X_e$=the effluent suspended solids concentrations .

$X_w$= the waste sludge flow rate.

$Q_r$= the return sludge flow rate.

If the solid are wasted from the reactor as shown in the dashed line the equation become :

$$C_c = \frac{XV}{Q_w X + (Q - Q_w) X_e}$$

If the system is operating, that is, if the clarifies is functioning the bulk of the solid will be removed in the waste. Sludge rather than in the effluent

$\theta_c$ can be controlled by varying $Q_w$ and is not completely depend upon the reactor volume.

A mass balance equation of the reactor :

$$\frac{1}{\theta_c} = \frac{Q}{V}\left(1 + r - r\frac{Xr}{X}\right)$$

R: is the ratio of the solid return flow to the raw waste flow $\frac{Q_r}{Q}$ .thus $\theta_c$ is function to both recycle ratio and $X_r/X$, the ratio of return solids concentration in the clarifies under flow to the mixed liquor solids concentration.

*law race and me arty showed that the total microbial mass in the reactor is give by:

$$XV = \frac{YQ(S_0 - S) * \theta_c}{1 + K_d\theta_c} \ or \ X = \frac{\theta_c}{\theta}\frac{Y((S_0 - S)}{1 + K_d\theta_c}$$

Y=the growth yield coefficient relating cell yield to the material metabolized.

$S_0$=the influent $BOD_5$.

S=the effluent soluble $BOD_5$.

θ= the liquid retention time in the reactor V/Q.

**Example :-** Design an actuated sludge process to yield an eff. $BOD_5$ of 20mg/ℓ and suspended solid of 25 mg/ℓ the influent $BOD_5$ following primary clarification 160 mg/ℓ, Assume Y=0.65, $K_d$=0.05, $\theta_c$ =10day, the waste flow is 10$m^3$/min.

Solution :-the $BOD_5$ of the effluent solid can be estimated to be 0.63 (s.s)

$BOD_5$ of the effluent reduced to 20-0.63 *(25)=4 mg/ℓ

Total biological mass is $VX = \frac{YQ\theta_c(S_0-S)}{1+K_d\theta_c}=\frac{0.56(10^4\times1440)(10)(160-4)}{1+0.05(10)} = 9.73\times10^9 mg$.

Assume mixed liguid volatile suspended solid (MlVSS) conc. 2500mg/ℓ (X)

$$V=\frac{9.73\times10^9\,\text{mg}}{2500\,\text{mg}/\ell}=3.894\times10^6\ell=3894m^3$$

The rate sludge production (biological solids) is

$$\frac{dx}{dt}=\frac{XV}{\theta_c}=\frac{9.73\times10^9}{10}=9.73\times\frac{10^8 mg}{d}=\frac{973.5Kg}{d}$$

If 80% volatile, total production $\frac{973.5}{0.8}=1217kg/day$.

The under flow solids conc. $X_r$=15,000mg/ℓ

$Q_w=\frac{1217\times10^6 mg/d}{15\times10^3 mg/\ell}$ =81.1$m^3/day$

Recirculation flow $Q_r.X_r=(Q+Q_r)X$

$$Q_r=\frac{Q\,x}{Xr-X}=\frac{10(2500)}{12500}=\frac{2m^3}{min}.$$

$R = \frac{Q\,r}{Q}$ =0.2=20%

or by use $1/\theta_c = \frac{Q}{V}\left(1 + r - r\frac{Xr}{X}\right) = 0.2$

the hydraulic retention time in the reactor is :-

t=V/Q=$\frac{3.89\times10^3}{1440} = 0.27day$ ⟶ 6.5hrs.

growth process is equal to the difference between the ultimate $BOD_5$ of the waste which is removed and ultimate BOD of the solid which are wasted.

For ordinary domastic sewage :

$Q_2$demand =1.47 $(S_0-S)Q-[1.14X_r(Q_w)]$

$Q_2$demand =1.47 $(160-4)14.4\times10^6-[1.14(15000(81,100)]$

= $1.915\times10^9$mg/d=1915 Kg/day.

Volume of air =$Q_{air}$= $Q_2$demand/0.232(1.2)

= $Q_2$demand/0.278

$Q_{air}$=1915/0.278=6856$m^3$/day

Actual air required =6856/0.07=97,943$m^3$/day≈ $\frac{68m^3}{min}$.

-the air volume required per unit $BOD_5$ removed

97.943/(160-4)14.4=43.6$m^3$/Kg

If X is high ⟶ smaller volume of reactor become X depend on $X_e$ and $Q_r$ ⟶ depend upon θc, $Q_2$ demand .

*see table 24-3 page.500 for more information

**Suspended growth process clarifier Design ;**

The designer should consider peak flow rate which is likely to enter the clarifier and its effect upon surface over flow rate , weir loading rate , and solid rating rate, the sludge handling equipment should be sized to recirculation up to 100 percent Q in order to provide for short term over loads.

Clarifiers may be rectangular or circular. Length or diameter of rectangular basin or circular basin does not exceed 10 times the depth .

Inlet baffles : see chap. 23. Horizontal velocity is limited to about 0.5m/min (1.5ft/min )in rectangular bas., the annular –inlet battles in center .fed circular tank should have a diameter of 15 to 20% of that of the tank it self.

Solids loading : is expressed in Kg SS/$m^3$ per hour.

$$\frac{\frac{Kg}{m^2}}{hr} = \frac{MLSS(\frac{mg}{\ell}) \times SOR(\frac{m}{d})}{24 \times 10^3}$$

2.5Kg/$m^2$/hr at average loading .

6.2 Kg/$m^2$/hr at peak loading.

See table 24-5 for SOR ,for MLSS conc. & recycle rate

Weir loading rate :<370 $m^3$/m/day (30,000 gal /ft/day )

(see conditions in page 515 )for tank depth 3.5m(12ft).

<250 $m^3$/m/day (20,000 gal/ft/day )for shallow basin.

The up ward velocity in the vicinity of the weir

<3to5 m/hr (10 to 15 ft/hr ).

**Example :-**Design a secondary clarifier for an activated sludge process with a recycle rate of 30 precut ,a MLSS conc. 3000 mg/ℓ ,peak flow 10,000 $m^3$/day.

Solution :- solid loading per hr. is

3000mg/ℓ×10×$10^6$ℓ/24 hr ×1.3=1625 Kg/hr

For peak flow loading is 6.2 Kg/$m^2$/hr

The area of basin =$A=\frac{1625Kg/hr}{6.2\frac{Kg}{m^2}/hr}=262m^2$

SOR=10,000 /262=38m/day

Depth use 3.5m , volume =Ax depth =262× 3.5 =917$m^3$

Detention time =$\frac{917}{10,000m^3/d\div 24}=2.2hr$

Use weir over flow rate 250 $m^3$/m/day

$$Length\ =\frac{10,000m^3/d}{250m^3/m/d}=40m$$

Use circular tank of 18.3m diam.

W. length =57m and area =263 $m^2$.

**Solid flux (coe. & clevenyer, yoshiska ete.):-**

Is the mass of solid per unit time passing through unit area perpendicular to the direction of flow.

In secondary clarifiers , it is the product of solids cancan. ration (mass/volume ) times the velocity (length /time ) units (Kg/$m^3$) times (m/hr) or (Kg/$m^2$.hr )

Down word velocity of . solids in a secondary clarifier has two components :

1- the transport velocity due to the with draw of sludge
2- Gravity settling of the solid relative to the water.

Transport velocity is a function of the under flow rate and the area of the tank $V_u = Q_u/A$ .

In clarifier the solid flux at given under flow rate is a linear function of the solids concentration $G_u = V_u X_i = (Q_u/A) X_i = (L^3/T/L^2) m/L^3 =$ mass / $L^2.T =$ flux is $G_u = V_g X_i$

$V_g$=settling velocity of solids due to gravity settling velocity decrease as solids conc. Rate increase in to the thickening zone.

In concentrated suspensions , conc. –velocity products will increase initially then in compression zone, gravity setting velocity be come insignificant and product of conc.-velocity $\longrightarrow$ zero.

The total solid flux is the same of the under flow transport and gravity flux $G_t = G_u + G_g$.

Limited by a minimum value from progressive gravity thickening. Also is determined by the under flow conc. $X_u$.

**<u>Example:-</u>** Design a secondary clarifier for activated sludge :-
A Column analysis was run to determine the settling characteristics of an activated .-sludge suspension.
The results of analysis are shown in table below :

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Conc. MLSS (mg/ℓ)** | 1400 | 2200 | 3000 | 3700 | 4500 | 5200 | 6500 | 8200 |
| **Velocity ( m/hr)** | 3 | 1.83 | 1.21 | 0.76 | 0.45 | 0.28 | 0.13 | 0.089 |

The influent conc. Of MLSS is 3000 mg/ℓ and the flow rate will thicker the solid to 10000 mg/ℓ, if Q=8000 $m^3/d$.

Solution:-

1- Calculate the solid flux.
   G=MLSS (Kg/m$^3$) × Velocity (m/hr)

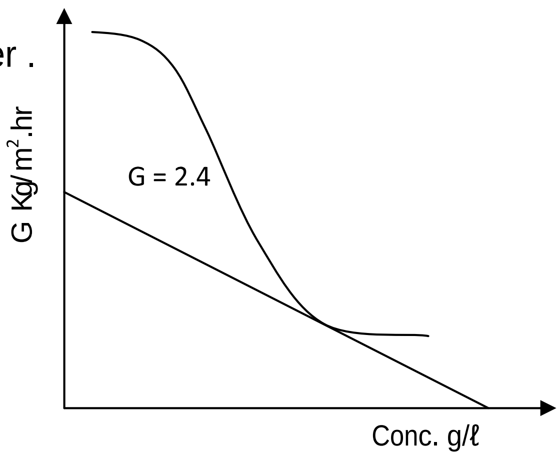
| Conc. MLSS (mg/ℓ) | 1400 | 2200 | 3000 | 3700 | 4500 | 5200 | 6500 | 8200 |
|---|---|---|---|---|---|---|---|---|
| G (Kg/ m$^2$. hr) | 4.2 | 4.07 | 3.63 | 2.8 | 2.03 | 1.46 | 0.9 | 0.73 |

2- Plot solid flux G , MLSS conc., draw line for conc. Sludge=10000 mg/ℓ tangent to the curve intersect ordinate G =2.4 is the limiting flux rate and governs thicken function.

3- Determine total solids loading to the clarifier.
   8000 m$^3$/d ×d/24hr×3Kg/m$^3$ = 1000 Kg/hr.

4- Determine surface area of the clarifier .
   1000 Kg/hr /2.4 Kg/hr.m$^2$ = 416.7 m$^2$
   Assume it circular shape
   $$Dia. = \left(\frac{4}{\pi} * 416.7\right)^{0.5} = 23m$$

5- Check clarification function .
   8000 m$^3$/d × d/24 hr =333 m$^3$/hr.




At 3000 mg/ℓ the settling velocity of interface is 1.21 m/hr.
333 m$^3$/hr /1.21 m/hr =275 m$^2$.

So use 416.7 m$^2$ the thicken function is governs the design.